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A Feasibility Study of Post-tensioned Stone for Cladding

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ABSTRACT

As contemporary building technology evolves to meet the challenges of increasing industrialisation and prefabrication of the construction process the demand for more efficiency also increases. Stone is a material of great architectural character and expression. It is used extensively in Scotland, in traditional building but now mostly in corporate or institutional buildings and indeed it helps define the character of many Scottish cities. It is however, not immune to the pressures for efficiency. Working in collaboration with a stone contractor a feasibility study was undertaken to examine ways to use stone more efficiently by reducing the quantity of stone and simplifying the installation process. This paper presents the preliminary results of a study into the structural behaviour and design of post-tensioned stone panels, intended as cladding for buildings. In contemporary applications stone is attached to buildings in a variety of ways, connecting to the supporting structure using steel angles and brackets or by incorporating the stone in a pre-cast concrete panel. By post-tensioning the stone, prefabricated panels can be made which eliminate either the secondary steelwork and subsequent site operations or avoid the need for pre-cast concrete panels. A series of full-scale post-tensioned beams using different thicknesses and types of stone were tested. The degree of pre-stress, the load-deflection response and serviceability of the beams is reported. Further studies were also carried out on full-scale prototype panels used to consider the construction process, anchorage details and methods of attachment to the primary structure. The results show that simple techniques for post-tensioning stone can be developed that reduce the need for highly skilled labour and can result in efficient construction processes. The results can be applied to a broad range of load bearing stone elements such as columns, beams and vaults.

KEYWORDS

Post-tensioned, stone, cladding.

1 INTRODUCTION

Natural stone is a material of great character and architectural beauty. Its roots lie in traditional craft based construction, in mass, load-bearing structures. However as framed buildings evolved in the 19th century it became used less for its physical qualities and more for its visual and symbolic attributes, as a skin to cover and protect the frame. The separation of the load-bearing function from the enclosing function of the wall enabled taller and lighter buildings to be constructed at faster rates. Various methods of attaching the stone to the frame have developed that rely on different levels of traditional stone masonry skills. The most common methods of attachment of stone to structural frames are:

- to use a secondary arrangement of stainless steel supports attached to a backup wall onto which the stone is hand-set using traditional stone masonry techniques and pointed using conventional mortars
- to construct pre-cast concrete cladding panels with stone facings (25 - 40 mm in thickness) mechanically attached during casting to a concrete panel
- Thin stone panels in an open jointed rain-screen, supported on a secondary framing system attached either to a back up wall or to secondary steel frame

These systems are covered by relevant British Standards, (1).

The construction industry is facing increasing demands to industrialise the fabrication and assembly processes, reduce material consumption and improve efficiency although current systems have some disadvantages. Stone compared with other cladding materials is relatively expensive and hence the use of stone is sensitive to these pressures. In all these approaches the stone is simply the outer layer of a multi-layer façade.. The use of hand set stones is slow and requires extensive scaffolding. The construction of a rain-screen façade requires considerable engineering design of the support framework and the use of pre-cast concrete adds additional weight to the structure. In all the systems the intrinsic compressive strength of the stone is not used, the primary structural action on the stone is flexure. An alternative construction is to use post-tensioning to create prefabricated stone cladding panels. These panels take advantage of the compressive strength of the stone and can eliminate either the secondary steel work and subsequent site operations or the need for pre-cast concrete panels, reducing both the overall weight of the panels and eliminating the need for formwork and concrete.

The paper presents a feasibility study into the construction and behaviour of post-tensioned stone panels. It was part of a larger project into the development of industrialised methods of stone cladding. An initial study was carried out on a simple panel to assess the proposed method of pre-tensioning. This was followed by a construction study to investigate the design and manufacture and installation of a full-scale panel without mortar. The final study was a series of structural tests on four panels of varying thickness and stone type to investigate the flexural behaviour and construction sensitivity.

2 PREVIOUS RESEARCH AND APPLICATIONS OF POST-TENSIONED STONE

There is relatively little published research on post-tensioned stone. During the second half of the last century there was, however, considerable research on the use of post-tensioned brickwork in both walls, Curtin (2) and in beams Pedreschi and Sinha (3). Without doubt however, the most sophisticated applications of this period can be seen in the buildings of Eladio Dieste, Pedreschi (4). The development paths of brickwork and stone, however have been quite different. Brickwork is seen as a low cost material that if reinforced and pre-stressed may offer a viable alternative to reinforced concrete, such as has been proven by Dieste. Stone on the other hand has a different association, use and application. It is generally more expensive and has been used most often in applications such as prestigious institutional or commercial buildings. The main published work has been on actual applications in buildings of architectural merit, such as the Queen's Building at Cambridge University, Werron and Dickson, (5). Three storey high limestone columns were constructed to carry the roof and floors of the building. The columns were also used to provide lateral stability to the building and were post-tensioned using stainless steel rods attached directly to the foundations. The columns were 800 by 400 millimetres in cross section at the ground floor reducing to 600 by 400 at the second storey. The pre-stress on each column was 250kN, applied via pre-cast concrete anchor blocks. The structural design was verified by a series of prototype column tests. A system of post-tensioned cladding panels was developed by the Indiana Limestone company, Donaldson (6), These panels were designed as spandrels in facades with continuous ribbon glazing. Panels spanning up to 9.0 between columns and 2.0 metres in depth were constructed. The panels were designed to carry both the self-weight of the stone and the applied wind loads. The Padre Pio Church, Foggia, Italy, Brown (7), uses post-tensioning in the construction of a series of 17 stone arches, which support the roof. The largest of the arches has a span of 45 metres. The arches were constructed using large blocks of marble (4.2 by 1.8 metres) bedded in epoxy resin mortar. More recently an office in Finsbury Square London, designed by Eric Parry Associates, was constructed using post-tensioned Portland stone columns. Each column is typically 3.8 metres in height and consists of blocks of stone with two 50 mm diameter holes to accommodate the 20 mm diameter steel rods. The columns were constructed and conventionally and left to set for 7 days prior to post-tensioning, Dernie (8). The same architect also used post-tensioned stone in the design of a tall, inclined needle. The 16 metre long needle was constructed from a series of 25 tapered Portland stone blocks. The blocks were triangular in cross-section and tensioned using stainless steel rods. Dernie (8). Perhaps one of the most adventurous uses of natural stone is the Inachus Bridge, located in Beppu, Japan, designed by Mamouru Kawaguchi. This lenticular shaped bridge uses Chinese granite only 25 centimetres in thickness as the post-tensioned compression chord forming a hybrid structure with a steel plate lower chord. The stone also acts as the walkway for the 34 metre span bridge (9).

3 INITIAL TESTS TO CONSIDER STRUCTURAL BEHAVIOUR AND CONSTRUCTION PROCESS

An initial study was carried out on a simple stone panel, to consider the feasibility of post-tensioning and study construction methodology. The panel was 0.5 metres wide by 1.86 metres long and 0.1 metres in depth. The panel was constructed using surplus sandstones from various sources, Clashach, Woodkirk and Cat Castle, from Scotland and the North of England. The stones were supplied in 500 by 300 by 100 blocks, with a 20 mm diameter hole drilled centrally. The panels were constructed vertically with 6 mm mortar joints in a simple stack bond arrangement. A series of trials mixes for the mortar were made before settling on a 1:6 mix of ordinary Portland cement and crushed Yorkstone sand. The pre-stress force was applied using a 16 mm diameter bar passing through the hole in the blocks and attached to steel anchor plates at each end. The threading of the rod through the blocks was quite difficult due to slight misalignments of the holes and some mortar falling into the holes during construction. Pre-stress was applied after three days. The bolts were hand tightened and the stressed using a spanner on the nuts at the end. The extension of the rod was carefully measured using a micrometer. An initial extension of 2.5 mm was applied, providing a pre-stress force of approximately 54 kN (1.08 N/mm^2).

Figure 1 Initial post-tensioned stone panel, 100 millimetres deep, with an effective span of 1.76 metres. Load is applied using stone blocks.

The panel was tested over an effective span of 1.76 metres between the centres of bearing. The prototype was constructed and tested in the banker yard of the stonemasons. A series of tests were carried out using stone blocks and clay bricks as loading. These were weighed carefully. Load was applied centrally to the panel in increments of 0.175 kN. A fine hairline crack appeared at a load of 0.88kN. On removal of load the crack closed and there was almost complete recovery of deflection. Figure 1 illustrates the panel under test.
Figure 2 illustrates the load – deflection response during the fifth cycle of load. The hairline crack in the central mortar joint had appeared in the earlier cycles. The panel was then loaded to twice the cracking load. Although the cracks opened further there were no signs of failure or distress in the stones themselves.

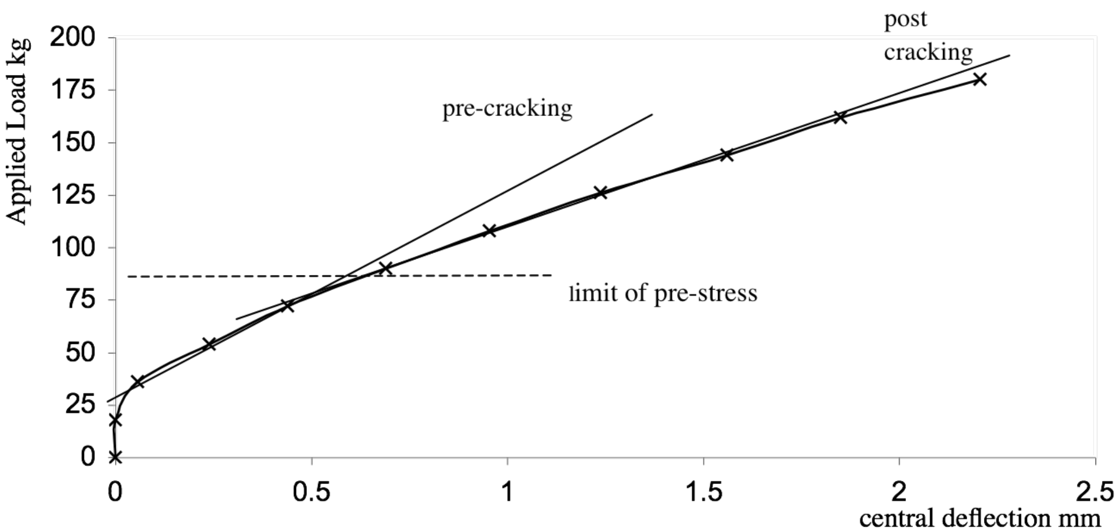

Figure 2 Load-deflection response for post-tensioned stone panel. The pre and post-cracking phases are identified

The panel demonstrated a high initial stiffness that was consistent in each load cycle, which may be due to some initial restraint at the bearings. Figure 2 also demonstrates the pre- and post-cracking phases of the load deflection response. There is a slight change in stiffness at the transition point. This occurs at the applied load of 0.88 kN. Using this load as the point of decompression of the pre-stress and taking into account the additional bending moment due to the self–weight of the panels then the applied pre-stress force was calculated as 50.5kN, close to the pre-stress force determined by measuring the extension of the rod. The pre-stress in the stone is 1.01N/mm^2.
This initial test demonstrated the feasibility of the post-tensioning, however, the construction process was slow and awkward, the test panel had to be constructed vertically, in two stages to allow the mortar set. The height of the panel also required scaffolding to lay the uppermost stones. The panel was also raised off the ground to allow

the anchor plates to be attached and braced in position braced for stability whilst the mortar set. These difficulties in the construction of the test panel would create significant logistical, handling and practical problems if scaled to a production run. An alternative approach was developed to simplify the assembly of the panels.

4. DRY JOINTED PANEL

The previous study confirmed the feasibility of the structural aspects of post-tensioned stone but highlight assembly and construction problems. The second study focused on practical aspects of manufacture and erection. If the mortar could be replaced with a dry joint significant production advantages could be gained.

- The panel could be assembled horizontally- removing the need to work at height and eliminating the temporary bracing
- The panel could be assembled much faster
- The problem of mortar filling the holes for the pre-stressing rod would be avoided.

It was felt important that the constructed panel should resemble traditionally constructed stone walls, therefore staggered vertical joints as well as horizontal joints were incorporated into the prototype.

Figure 3 Assembly of panels

Figure 4 Prototype post-tensioned stone panel erected on test rig

The panel was constructed using 100 mm thick stone blocks. The panel was arranged in a staggered bond arrangement, typical in many façades. The joints between the blocks were formed using 5 mm polycarbonate sheets, cut and drilled to receive the pre-stressing rod and placed at each horizontal and vertical joint. The polycarbonate sheets were cut back from the face of the stone to allow pointing using coloured silicone and stone dust, a technique the stone contractor had developed for a previous project as a repair to joints that had cracked due to transient traffic vibrations. Key elements of the design were the anchor plates for pre-stressing and the fixing brackets to attach the panels to the primary structure In actual applications it would not normally be visually acceptable to have the head of the bolts exposed at the ends of the panels. Fabricated steel details were developed to combine the functions of the pre-stressing anchor plates and the connections to the primary structure in a single component such that as soon as the panels was tensioned it could be lifted. The brackets were also designed to run the width of the panel and connect the steel tension rods together. Two 16 mm diameter steel rods were used to apply pre-stress. Again 20 mm diameter steel holes were drilled in the blocks to receive the rods. To avoid rotation of the blocks around the vertical joints the stones were connected using six mm diameter steel dowels. Rebates were cut in the top and bottom stones of the panels to receive the fabricated brackets and hide the ends of the steel tensioning rods. The details were developed that enabled these to be hidden on the façade. The panels were assembled on a horizontal table with the blocks and the polycarbonate spacers and dowels in place and then post-tensioned in the same manner as before. Assembly of the panels and fixings and post-tensioning was completed in less than two hours. The panels were then lifted from the table into a vertical orientation by overhead crane and then installed on a steel frame test rig. The self-weight was carried by the bracket at the base of the panel and the top used to provide lateral restraint. The panel was installed easily on the rig by crane with no difficulty, figure 4. The combined bracket and stressing anchor is shown in figure 5. The study demonstrated quite effectively the considerable improvements possible using dry construction methods. The completed panel could be installed directly onto the building from its delivery vehicle with the connections being made from inside the structure, thus eliminating the need for scaffolding with traditional hand-set methods.

Figure 5 Combined connection and post-tensioning anchorage

5 STRUCTURAL TESTS ON POST-TENSIONED DRY STONE PANELS

Having established the construction methodology and the viability of the dry assembly process a further series of structural tests was carried out to study the behaviour of the beams and the effect of the polycarbonate joint. Four post-tensioned beams were tested using two types of sandstone in three thicknesses. A series of material tests was carried out to determine the physical properties of the stone.

5.1 Material properties of stone

The two types of sandstone used were Clashach and Stokehall. Clashach comes from a quarry in the coast of the Moray Firth in the North East of Scotland and is a hard and very durable stone. Stoke Hall comes from Derbyshire in the Midlands of England and whilst durable and strong is considered less so than Clashach. Both stones were chosen as being representative of the types of stone specified for the facades of buildings in Scotland. Both had been used in recent projects in Edinburgh.

5.1.1 Compressive strength

Tests were carried out on both stones in accordance with BS EN 1926, (10), Ten 70 mm cubes were tested perpendicular to the bedding plane and ten parallel to the bedding plane. The results are summarised in table 1

Table 1 Summary of compressive strength test on stones used in the construction of the test beams

Stone type	Direction of applied load to bedding plane	Average compressive strength MPa	Maximum compressive strength MPa	Minimum compressive strength MPa	Standard Deviation MPa	Coefficient of Variance %
Clashach	perpendicular	141.2	183.7	83.7	32.3	22.8
Clashach	parallel	139.4	180.0	72.2	40.9	29.3
Stoke Hall	perpendicular	88.0	100.8	73.5	8.9	10.1
Stoke Hall	parallel	85.9	108.2	73.5	10.3	12.0

The Claschach stone is clearly considerably stronger that Stoke Hall although there is a greater variation across the test samples. The results are compared with figures reported by the Building Research Establishment, (11). The reported compressive strength perpendicular to the bedding plane is 132.2 MPa, for Clashach and 103 MPa for Stoke Hall stone.

5.1.2 Flexural Strength

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Tests were also carried out on the flexural tensile strength of the stone. Twelve samples of the Clashach and twenty samples of Stoke Hall were tested in accordance with BS EN 12372, ((12). The results are summarised in table 2

Load arrangement

Table 2 Summary of flexural tests on stone

Stone type	Average Flexural strength MPa	Maximum flexural strength MPa	Minimum Flexural Strength MPa	Standard deviation MPa	Coefficient of Variance %	Average Modulus of elasticity GPa
Clashach	15.2	16.9	10.9	1.57	10.3	10.7
Stoke Hall	8.9	10.1	7.5	0.68	7.6	4.8

Again the Clashach has considerably greater flexural tensile strength than the Stoke Hall stone.

5.1.3 Properties of the polycarbonate.

The polycarbonate sheet was approximately 5.9 mm in thickness. Samples were tested in flexure and in tension. The average modulus of elasticity from flexural tests was 2.21 GPa, considerably lower that either of the two stone types, and average tensile strength was 46 MPa.

5.2 Construction and post-tensioning of the test panels

Four test panels were constructed. Three were built using Clashach stone in different thicknesses, 75, 100 and 125 mm. and one using Stoke Hall, 100 mm in thickness. The stone supplied and cut by Watson Stonecraft, pre-cut into blocks 320 mm by 300 mm by the thickness. Twenty millimetre diameter holes were drilled through the centre of each block using a wet diamond core drill to receive a 16 mm diameter steel post-tensioning bar. The steel bars were threaded at each end. As there was only one tensioning rod, two 6 mm diameter steel dowels were incorporated between each block to prevent rotation of the blocks. The dry joint spacers were cut from 6 mm polycarbonate sheet into 320 by 75, 100 or 125 plates, depending on the thickness of the test panel and drilled to accommodate both the 16 mm tensioning rod and the 6 mm steel dowels. The test panels were assembled on steel rails on the floor of the Architecture workshop at the University of Edinburgh to ensure a level surface. The stone blocks and polycarbonate spacers were threaded onto the steel rods sequentially. When the full beam was assembled 8 mm thick steel end plates were added at each end. Spring washers were installed and nuts were threaded onto the rod. The nuts were tightened till the washers compressed and the joints between the blocks were closed. This was taken as the datum for the start of the pre-stressing. A nominal pre-stress force of 52 kN was applied The force was determined by carefully measuring the extension of the rod using a micrometer.

5.3 Test arrangement and procedure

The stone panels were installed on a test rig and subject to four point loading, illustrated in Fig. 6.

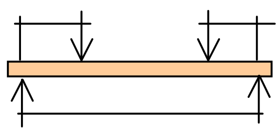

Figure 6 Test rig and loading arrangement for stone panels

Load was applied using a hydraulic ram and spreader beam. Deflections were measured using dial gauges at mid-span and at the supports to allow for any movement of the rig. Demec points were placed on the surface of the stone panels at various positions to measure the strains in the stone and across the joints in the panels.

The panels were subjected to 5 cycles of load:

- 1st cycle – 80% of load to cause decompression of nominal pre-stress force
- 2nd cycle – 100% of load to cause decompression of nominal pre-stress force
- 3rd cycle – 100% of load to cause decompression of nominal pre-stress force
- 4th cycle – 130% of load to cause decompression of nominal pre-stress force
- 5th cycle – 200% of load to cause decompression of nominal pre-stress force.

It was assumed that in normal practical applications the load to cause decompression of the pre-stress force would determine the maximum applied working load caused by wind. Load cycles 2 and 3 are therefore working load levels. Cycles 4 and 5 represent overload conditions.

5.4 Results and Observations

The test results are summarised in table 3. The pre-stress force was estimated from the load deflection behaviour during the 4th and 5th cycles of load. As the load applied to the panels exceeds the load required to cause decompression of the pre-stress there is a change in slope of the deflection curve.

Table 3 Summary of experimental results on stone panel tests

Beam	Pre-stress force measured by extension kN	Pre-stress force determined from deflection kN	Pre-stress in stone MPa	Flexural stiffness EI x 10^{-9} Nmm^2	Recovery of deflection % of max. deflection in 5th cycle
75 mm Clashach	53.7	49.5	2.1	74	96
100 mm Clashach	52.3	46.2	1.4	196	100
125 mm Clashach	52.6	41.6	1.0	256	97
100 mm Stoke Hall	51.7	43.4	1.4	33	92

The load at which decompression occurs is determined by linear regression analysis of the deflection data, Fig.7 This is illustrated for the 5th load cycle of the 100 mm thick Clashach stone panel. The pre-stress force was determined from the average decompression load of 4th and 5th cycles with the exception of the Stoke Hall panel, which was based on the 5th cycle only.

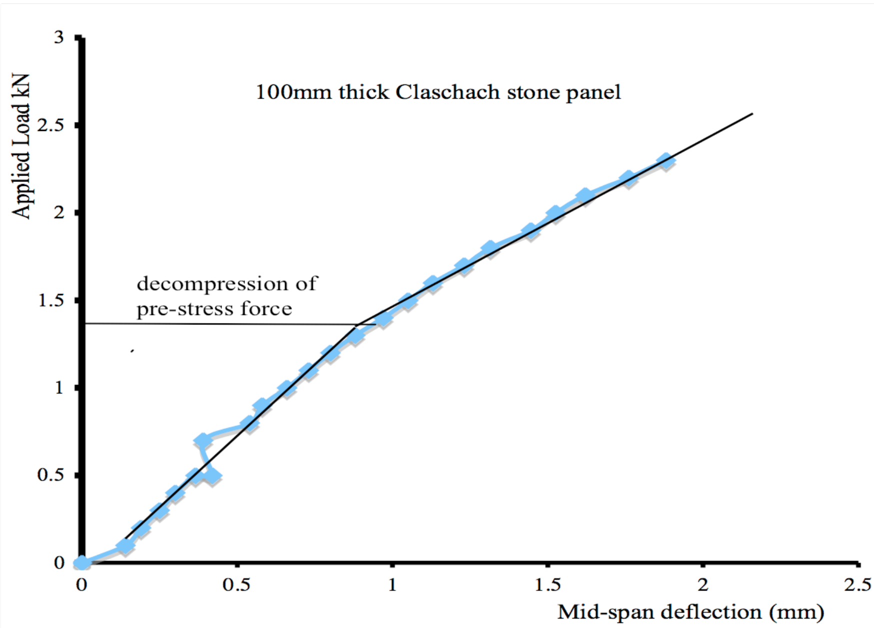

Figure 7 Load – deflection response for 100 mm thick Clashach stone panel for the 5th cycle of load.

The pre-stress forces obtained from the load-deflection results are less than the nominal pre-stress, on average 13%. During the pre-stressing operation in was noticed that some of the panels developed a slight curvature,

attributed to variations in the dimensions in the blocks, see Fig.8. Estimating the pre-stress force by measuring the extension of the rod is very sensitive to the datum position from which the measurements are taken. If all the panels are laid horizontally and the initial slackness is removed by tightening the rod then this is assumed to be the datum position for measurement of pre-stress. However if the blocks are not fully in contact for the full depth of the joint then application of the pre-stress force will cause the blocks to rotate slightly and this will lead to an apparent increase in extension and would account for the difference in the pre-stress force determined by each method. The rotations of the block during pre-stressing will tend to shift the position of the line of action of the pre-stress. This will occur as the blocks move relative to the steel rod. The maximum relative movement is ± 2.0 mm assuming that 16 mm diameter rod is initially central to the hole in the blocks. The rotation is caused by the ends of the blocks aligning with each other. Once aligned the force across the joint should be essentially uniform as the possible range of eccentricity is small. It is also interesting to note that this type of behaviour was not observed in the larger scale prototype panels. The variation in thickness meant that the strain distributions measured after pre-stressing were erratic as they record both strain and joint rotation.
All test panels demonstrated almost 100% recovery of deflection after each load cycle, table 3. The effect of depth and stone type can be seen clearly in Fig. 9. Comparing the Stoke Hall panel with the 100 mm Clashach panel, although the panels have the same nominal dimensions the Stoke Hall panel is considerable less stiff, at the load to cause decompression the deflection is over five times greater. The influence of depth of panel is also marked, see also table 3.

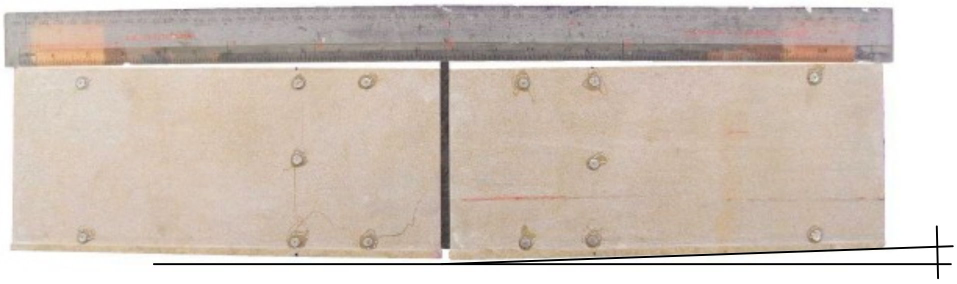

Figure 8 Curvature of blocks due to variations in block dimensions

As would be expected the 125 mm thick panel is stiffest, 30% greater stiffness than the 100 thick panel and approximately 350% stiffer than the 75 mm thick panel. The increase in stiffness is not, however in proportion to the cube of the depth of the panels, as conventional theory would suggest, but considerably less. It should be noted that the value of flexural stiffness, EI, in table 3 is a composite value derived experimentally and includes the effect of the polycarbonate spacer.

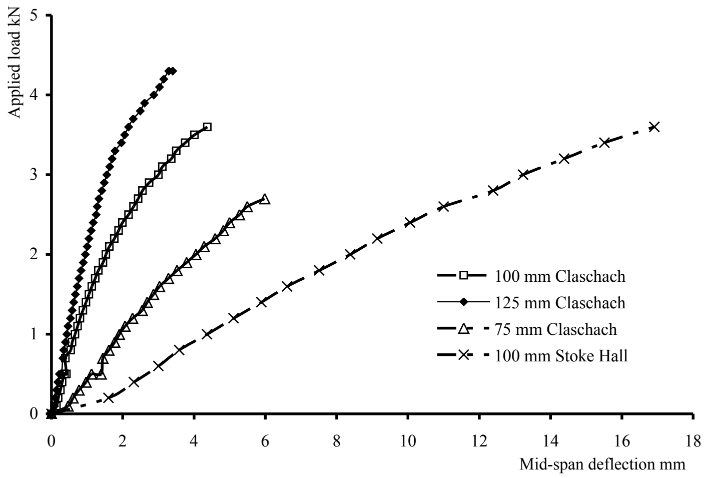

Figure 9 Load-deflection response for 5^{th} cycle – all test panels

All panels were loaded past the point of decompression and although there was an increase in the rate of deflection and the joints between the panels were opening visibly, there were no signs of imminent failure at twice the load to cause decompression. By correcting the strain measurements, setting the datum from the position of zero applied load a qualitative picture of the development of the strain cross the section of the panel can be seen. This is illustrated in Figure 10 for the 100 mm panel using Clashach stone. As the load increases

and the initial pre-compression is neutralised there is a shift upwards in the position of the neutral axis depth and is consistent with the reduction in flexural stiffness illustrated in Figure 7.
The influence of the polycarbonate spacer on the stiffness is illustrated in table 4. Here the flexural stiffness based on the elastic modulus of the stone itself and the nominal cross-sectional dimensions has been calculated. From the results it can be seen that the influence of the polycarbonate is quite significant. The flexural stiffness is reduced to between 26 and 69 % of the stiffness based the stone alone. The panels using Stoke Hall stone showed the greatest reduction in stiffness.

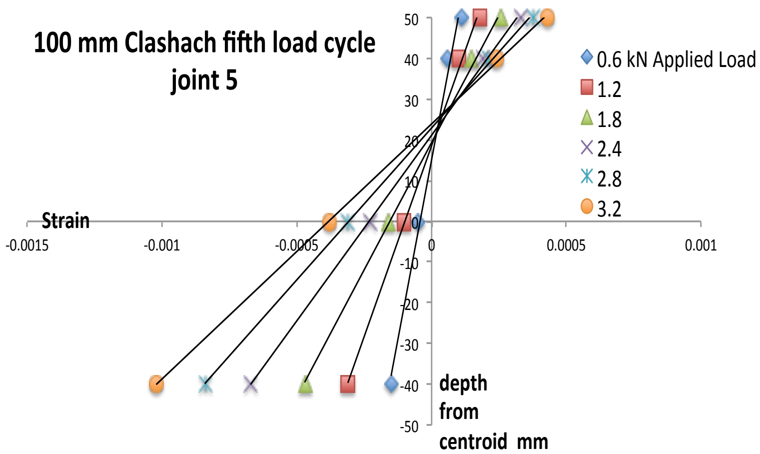

Figure 10 Strain distributions through depth for 100mm thick Clashach panel

Table 4 Comparison of EI of the stone accounting for influence of polycarbonate

Stone type	**I (mm x 10^6)**	**E (GPa)**	**EI theo Nmm^2**	**EI experimental Nmm^2**	**EI exp/EI theo**
Clashach 75	11.25	10.7	120	74	0.62
Clashach 100	26.7	10.7	285	196	0.69
Clashach 125	50.1	10.7	557	256	0.46
Stoke Hall 100	27.7	4.8	128	33	0.26

Comparison with the earlier initial test, constructed using conventional mortar is quite interesting. The test sample was 100 millimetres in thickness. The flexural stiffness, prior to cracking, corrected to a width of 320 millimetres was 60.1 x 10^9 mm^2 compared with 196 and 33 x10^9 for the 100 mm deep panels using Clashach and Stoke Hall stone respectively. Both the depth and the spacing of joints was the same for all beams. The initial test panel was constructed with a variety of available stones. The Woodkirk and Cat Castle stones share a similar petrographic history to Stoke Hall, being geologically earlier coarse grained gritstones compared with the finer grained mudstone Clashach. The stiffness is clearly less than the stiffness of the stone itself (assuming properties similar to Stoke Hall. The interaction between bedding layers and brick or stone is complex, with relatively little research. Hendry (13) reported on tests on the compressive strength of brick masonry using differing bed joint materials including mortar, steel and rubber. Thin steel increases the strength brick assembly by more than 40% of the strength of the brick itself whilst rubber joints resulted in a 87% reduction in strength. More recently (14) tests on sandstone have shown that the stiffness of sandstone in axial compression is strongly affected by the nature of the bed-joint, with dry joints exhibiting greater stiffness than mortar joints.
The significance of the reduction in flexural stiffness of the panel due to the incorporation of the polycarbonate spacer can be checked by considering the maximum deflection of a panel in a realistic application. Consider a panel 1.0 metre wide spanning 3.0 metres between floors subjected to a maximum wind pressure of 2.0 kPa. Assuming the panel is simply supported and is constructed using 100 mm thick Clashach stone. The EI value from tests is based on a 0.32m wide panel and is multiplied by 1/0.32 to convert to a metre wide panel. The maximum deflection d is:

$$d = \frac{5wl^4}{EI} = \frac{5.2.0.3000^4}{384.196.10^9.3.125} = 3.44\text{mm}$$

This is a span to deflection ratio of 872. Although there are no current Standards for pre-fabricated post-tensioned stone panels BS 8298 part three (1) suggests a limit of span /360, for stone faced pre-cast concrete. On this basis the deflection due to wind is therefore well within allowable limits. On the other hand a similar beam using Stoke Hall would deflect 20.4 mm, span to deflection ratio of 147 and may therefore need to be constructed using thicker stones in order to reduce the deformation.

6 SUMMARY AND CONCLUSIONS

A series of preliminary studies have been undertaken to assess the feasibility of pre-stressing in the use of stone on building facades. The structural tests on the panels indicated the following:

1. The panels showed elastic behaviour up to the point of decompression of the pre-stress force with an almost a complete recovery of deflection.
2. Overloading the panels past the point of decompression resulted in a reduction in flexural stiffness as the joints opened up. However the joints closed on removal of load.
3. All panels were overloaded to twice the decompression load of the pre-stress without signs of failure or permanent deformation.
4. The incorporation of a dry polycarbonate spacer between the stones reduced the flexural stiffness of the panel to between 69 and 26% of the flexural stiffness of the stone assuming the panels were solid stone with filled joints.
5. The type of stone has a significant influence on the flexural behaviour of the panels and further study is needed to clarify this effect.

The techniques used in the production of the panels are straight-forward and the levels of pre-compression needed are very low in comparison to the compressive strength of the stone, effectively utilising the primary strength of the material. Considerably higher levels of pre-stress could be applied, although it would be prudent to study creep and loss of pre-stress. This study has not considered durability and in-keeping with similar environmental conditions it would be necessary to use stainless steel. In comparison with traditional hand set stone the need for intermediate restraint and a structural backup wall is also eliminated. The technique has been shown to be suitable for the construction of non-load-bearing building facades. The replacement of mortar with a dry joint simplifies the construction process by eliminating the time needed for curing and allowing the panel to be constructed horizontally and subsequently installed directly onto the primary frame. However accuracy of the cutting of the stones is more critical than in conventional stonework with mortar joints and closer tolerances are required when using dry assembly methods. Although not part of this study the dry methods of construction would also facilitate deconstruction and re-use of the stone at end of building life.

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